

GEOMETRICAL NONLINEARITY ON COLLAPSE OF REINFORCED CONCRETE PIERS

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The objective of this study is to investigate the possible collapse mechanisms of reinforced concrete columns. Analyses are performed to study the effect of both material and geometrical nonlinearities in the post peak response of flexural columns. Discussion is mainly focussed on the restoring force characteristics, especially in large deformation range, of RC columns having high shear capacity so that the ultimate failure is governed by bending.

Key Words: *geometrical nonlinearity, nonlinear analysis, cover spalling, reinforcement buckling, collapse*

1. INTRODUCTION

Reinforced concrete piers might be subjected to unexpected overload during a major earthquake. In such cases, the RC piers may exhibit one of the following two behaviors. If the design failure mode of the pier is shear, it may completely collapse showing brittle shear failure along a diagonal shear crack as the imposed shear force due to the unexpected earthquake becomes higher than the shear capacity of the pier section. On the other hand if the pier is designed to avoid shear failure, the residual deformation becomes considerably high due to the high plasticity developed in the reinforcement. In the later case, P-delta effect due to the weight of superstructure may govern the stability of the pier.

In seismic design, the ground motions are classified according to the value of maximum ground acceleration and their probability of occurrence. The requirements of seismic performance expected in structural system are defined relative to the level of seismic motion and importance of the structure. The code provides restricted plastic displacement to the structure so that in spite of the residual deformation due to an earthquake, the structure can be reused after basic repair. To protect human lives and to avoid the complete collapse of the structure for huge earthquake with very long return period, sufficient ductility is required. For example, seismic performance level 3 of Japan Society of Civil Engineers (hereafter referred to as JSCE) seismic

design code¹⁾ can be referred.

To achieve the aforementioned goal, the structures are designed to fail in flexure by ensuring the shear capacity higher than the bending capacity. However, there still remain some unanswered questions, such as: Does avoiding shear failure completely rule out the possibility of collapse of structure? Can collapse be prevented merely by ensuring bending failure mode? Is not there any collapse or instability of structure in bending?

The shear capacity of any RC structure V_{yd} consists of two parts, shear contribution of web reinforcement V_s and shear contribution of concrete V_c . The shear contribution of web reinforcement is calculated by the conventional truss analogy and that of concrete is calculated by the corresponding empirical equations incorporated in the design codes. Previous researchers^{2, 3, 4)} have verified that the contribution of concrete in shear capacity decreases with increase in deformation or damage level. However, as shown in **Fig.1**, if the structure is designed so that the shear contribution of transverse reinforcements is larger than the bending capacity V_{mb} , shear failure can be avoided no matter how large the damage level is. Of course, the shear capacity further decreases after the breaking of the transverse reinforcement, but it corresponds to very high deformation level that rarely takes place in real loading. Hence if designed properly, a structure can be forced to avoid brittle shear failure and to undergo desired ductility level.

JSCE seismic design code¹⁾ specifies that the failure mode should be checked by comparing shear strength of the member V_{yd} with the maximum shear

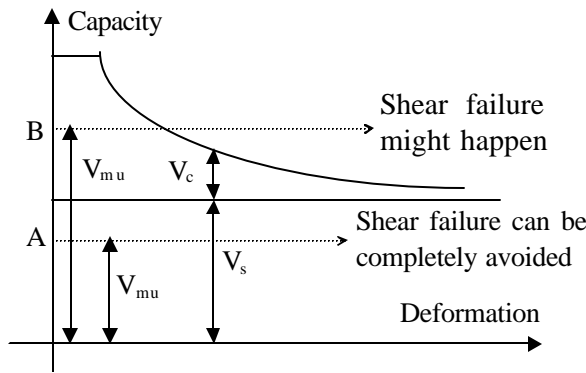


Fig.1 Determination of failure mode

force V_{mu} acting on the member when the bending moment at critical section reaches the flexural resistance of the member. The aforementioned provision of JSCE seismic design code can avoid shear failure before the yielding of the reinforcement, but there still remains the possibility of post-yielding shear failure due to the decrease in concrete contribution as the damage level increases. Dhakal and Maekawa⁵⁾ analytically verified that such structures can undergo shear failure before sufficient ductility is achieved and energy dissipation capacity, which is important for seismic design, is also significantly affected. Moreover, through the study of failure mechanisms of bridge piers failed during Hanshin Earthquake, Kim⁶⁾ concluded that possibility of shear failure still exists if the ratio of V_{yd}/V_{mu} is less than 1.3.

Apart from this, JSCE seismic design code¹⁾ also specifies that a response ductility factor of approximately 10 can be ensured if V_{yd}/V_{mu} is equal to or larger than 2 and no special consideration is required. This provision seems to be based on the assumption that at least half of the shear capacity comes from the shear reinforcement. If this condition is not satisfied, such a high ductility cannot always be ensured. Because of additional shear contribution from concrete, Tanabe⁷⁾ found that the columns designed by JSCE seismic design code are provided with fewer amounts of lateral reinforcements compared to those designed by other codes. A more rational recommendation to confirm the failure mode and to ensure the required ductility should be either to explicitly consider the decrease in shear contribution of concrete V_c or to increase the safety factor used in the comparison between V_{yd} and V_{mu} . This will probably increase the amount of lateral reinforcements, which helps to improve the structural performance in three ways. First, it increases the shear capacity and avoids shear failure mode. Second, it provides more confinement to core concrete. Last, but not least, it will improve the post-peak behavior by restraining the longitudinal reinforcements against buckling.

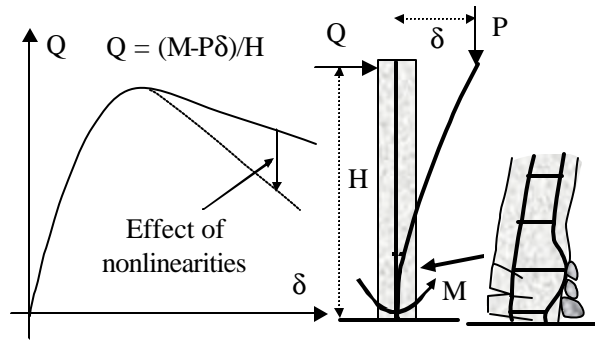


Fig.2 Effect of nonlinearities in post-peak response

2 . GEOMETRICAL NONLINEARITIES

Two-dimensionally modeled members usually fail either by crushing of concrete or by a diagonal shear crack. The maximum post-peak response of such structures is usually not so large. Obviously in such cases, analytical prediction with proper consideration of material nonlinearity exhibits sufficient agreement with the experimental results. On the other hand, flexural beams and columns can be loaded without failure until the post-peak deformation becomes significantly large. Dhakal and Maekawa⁸⁾ reported that the response of such structures, especially in high deformation post-peak region, is over-estimated if the geometrical nonlinearities associated with the system are overlooked. By geometrical nonlinearities, the authors mean the combination of P-delta effect and the local nonlinearities associated with the inelastic material behaviors, such as cover concrete spalling and large lateral deformation of reinforcement, hereafter referred to as buckling. As post-peak response and geometrical nonlinearities are not important issues for structures prone to shear failure, the discussion hereafter is focussed only on the post-peak behavior of flexural columns.

The effect of geometrical nonlinearities in the post-peak response of flexural column is illustrated in Fig.2. It can be noticed that the P-delta effect causes the reduction in restoring force characteristics of flexural columns and this difference becomes more significant with the increase in axial load levels; i.e. weight of superstructures in real columns, and also with the increase in lateral displacement. Moreover, the inelastic response of such flexural columns are accompanied with the spalling of cover concrete and reinforcement buckling in the compression side of the plastic hinge region.

The reinforcement under high compression bends laterally outwards, especially in the plastic

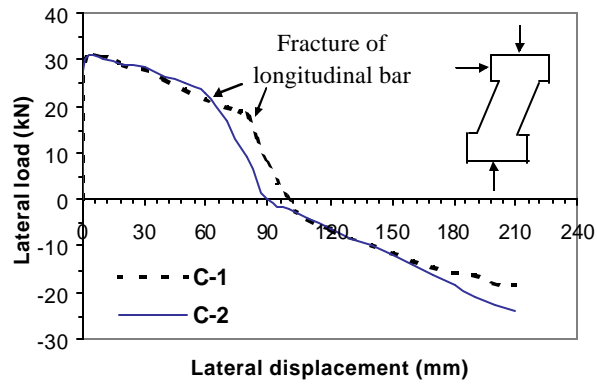
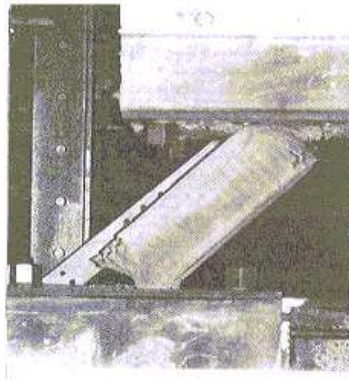


Fig.3 Collapse tests of RC columns by Takiguchi⁹⁾

hinge region, due to the average compressive stress of the reinforcement which shows softening behavior unlike hardening in tension. Moreover, the cover concrete, which does not have any external confinement, loses its load carrying capacity due to the combined effect of high compression and the lateral deformation of the reinforcing bars. Nevertheless, the core concrete is confined by the longitudinal and lateral reinforcements and hence can maintain its mechanical performance until very high deformation is reached. These local and inelastic material mechanisms also influence the post-peak response of flexural columns.

3. FLEXURAL INSTABILITY

In order to completely avoid the possibility of collapse, the stability of RC flexural columns under high axial compression should be properly checked, especially in high displacement range. In other words, merely avoiding shear failure mode may not always be sufficient to ensure the seismic performance level 3. Of course, experimental study comprising of collapse tests of axially loaded columns subjected to high lateral displacements is the direct and most reliable way to obtain related facts. However, it is extremely difficult to conduct such experiments because significantly high displacements are required to be applied, which might create safety problems. This might be the reason why most experiments are terminated before the columns become unstable.

The small-scale experiments conducted by Takiguchi et al.⁹⁾ should be mentioned here. Special loading arrangements were prepared with oil jacks to properly apply high deformation and to cope with the safety requirements. To ensure extremely high damage level within the allowable loading range of the setup, very small specimens were used. Here, two of the specimens *C1* and *C2* are referred. The

column height is 30cm and the cross-section is 10*10cm with 4mm clear cover. Four 6mm-diameter bars with yield stress 374MPa and yielding strain of 0.19% are used as main reinforcement. Similarly, 3mm-diameter bars at the spacing of 15mm with yield stress 653MPa and yielding strain of 0.52% are used as lateral ties. Axial compression equal to 51kN is applied, which corresponds to 9.4% and 8.5% of the axial capacities, given the concrete compressive strengths are 54.2MPa and 59.5MPa respectively.

The results of collapse tests of these two specimens are depicted in **Fig.3**. As the specimen size is very small compared to real columns, it is not logical to come up with some quantitative conclusion from these results. However these results provide ample proof that RC columns can become unstable even in bending. Breaking of longitudinal reinforcement could be observed in the experiments, and more interestingly, the lateral restoring force became negative after a high displacement was applied. It is believed that the structure becomes unstable once the restoring force becomes negative and breaking of reinforcement helps a lot to cause this.

From **Fig.2**, it can also be clearly understood that if the overturning moment induced by P-delta effect becomes higher than the sectional bending capacity corresponding to the current damage level, a lateral load (support) from opposite direction is required to stabilize the column at current deformation state. It is believed that if the restoring force characteristic of the column is already negative and the column is not provided with any lateral support from opposite direction, the excessive overturning moment due to P-delta effect will render the column unstable. Obviously, complete collapse in such situation cannot be avoided. This behavior is investigated in more detail hereafter.

The flexural resistance of a reinforced concrete section is calculated considering the contribution

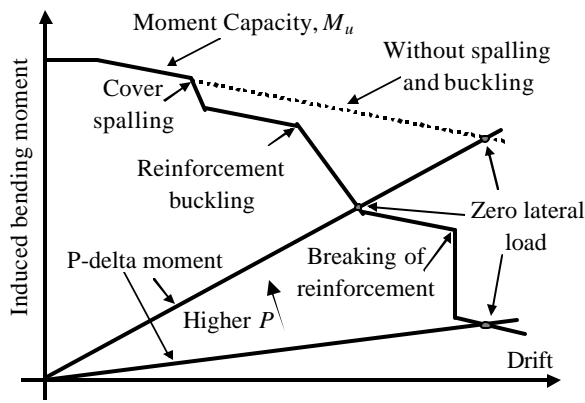


Fig.4 Reduction of flexural resistance in high deformation

from concrete and reinforcement in compression and reinforcing bars in tension assuming the tensile strain at the reinforcement is equal to yielding strain. This method gives a fair prediction of bending capacity until yielding of longitudinal bar. Nevertheless, as the deformation level increases and the structure enters in post-yielding region, the bending capacity slowly decreases due to the compression softening of concrete. As shown in **Fig.4**, the capacity is further reduced because the cover concrete loses its load-carrying capacity due to spalling and average compressive stress of reinforcement decreases due to buckling. In very high deformation range, this behavior is further accelerated due to the breaking of reinforcement.

It can be distinguished from **Fig.4** that if spalling and buckling are overlooked, the post-yielding flexural response of RC column is overestimated. It can be noticed that if the overturning moment induced by P-delta effect becomes equal to the sectional bending capacity corresponding to the current damage level, the lateral restoring force becomes zero. The drift at which restoring force becomes zero depends on the amount of axial load and the initial bending capacity of the section, which depends on the reinforcement ratio, geometrical properties of the section as well as the mechanical properties of the materials used.

4. ANALYTICAL APPROACH

As mentioned earlier, it is difficult to conduct proper experiments to obtain quantitative information in very high deformation range. Hence, the only alternate is to perform analytical study. The analytical tool should include proper material constitutive laws that implicitly or explicitly include the inelastic mechanisms of the constituent materials such as spalling of cover concrete, buckling and breaking of reinforcement as their effect in high-

deformation flexural behavior of RC columns cannot be neglected.

For the analytical prediction of collapse mechanism of RC piers, fiber technique^{10, 11)} is adopted in this study. In fiber technique, each element is represented using a single line coinciding with the centerline of the member. The member cross section is divided into many cells or sub-elements. The strain of each cell is calculated based on the Euler-Kirchoff's hypothesis, i.e. plane section remains plane after bending. For each fiber strain along the axis of finite element, response is calculated using the material constitutive models representing the average behavior. As is well known, the overall response of each element is the integrated response of these fibers and the overall response of the member comprises of all the element responses.

In fiber technique, the stress field is reduced to one dimension along the axis of finite element or members. Then, the shear force is computed so that it satisfies the equilibrium with flexural moment field and the out-of-plane shear failure is not inherently captured due to degenerated formulation of stress field for simplicity. However, in-plane shear deformation is considered based on Timoshenko's beam theory. As linear in-plane shear behavior is assumed in the analysis used in this study, inelastic shear deformation cannot be captured. Conclusively, if the shear strength of the concerned structure is high enough to ensure flexure failure so that the inelastic shear deformation is not so high, the performance of fiber technique is proved to be sufficiently reliable. As this study mainly concerns with the flexural behavior of columns with higher shear capacity, it is believed that the linear shear deformation assumption adopted in the formulation will not have much influence on the predicted results.

The schematic representations of fiber technique and the constitutive models used for concrete and reinforcement in each fiber are shown in **Fig.5**. In order to incorporate cover spalling, the concrete cells in the column cross-section are divided into two parts; i.e. cover concrete (concrete cells outside the longitudinal reinforcing bars) and core concrete (concrete cells inside the longitudinal reinforcing bars). The compressive behavior of concrete fibers is computed by elasto-plastic and fracture model¹²⁾, but the stress carried by cover concrete fibers is completely released once the spalling criteria is met. The spalling of cover concrete fibers is assumed to occur due to the combined effect of cracks due to compression of cover concrete itself and, more importantly, the lateral thrust due to the buckling tendency of the nearby longitudinal reinforcing bars.

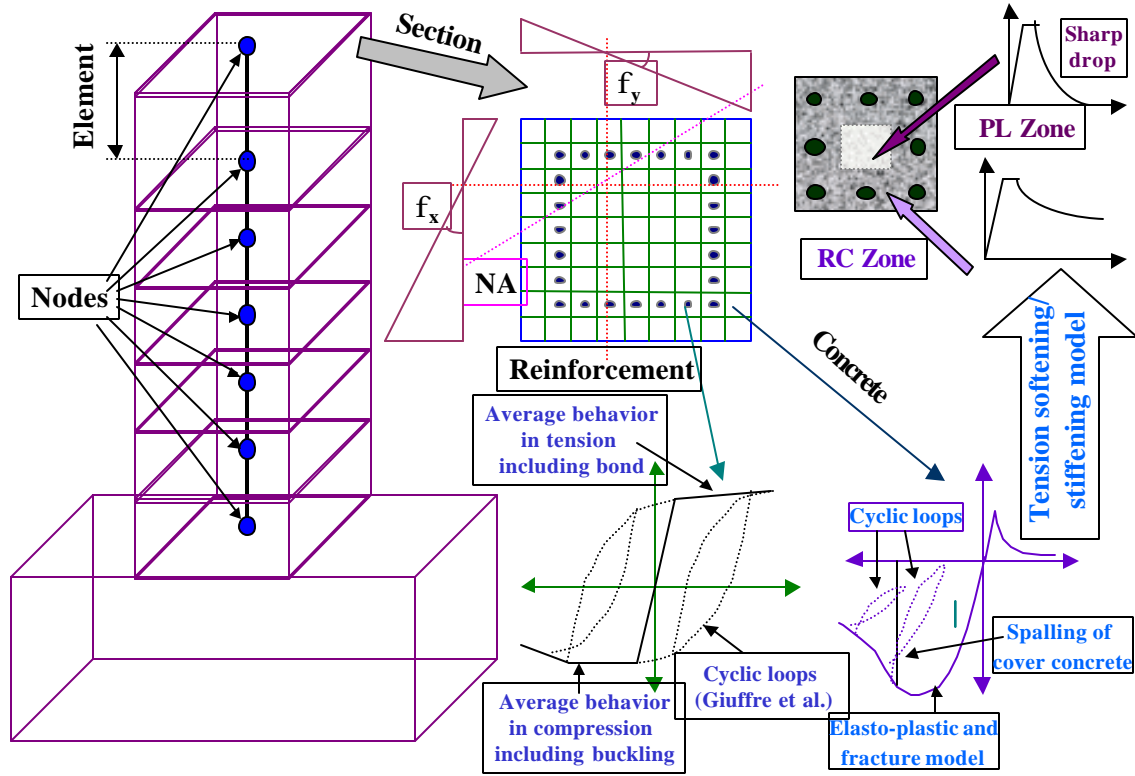


Fig.5 Fiber technique and constitutive models.

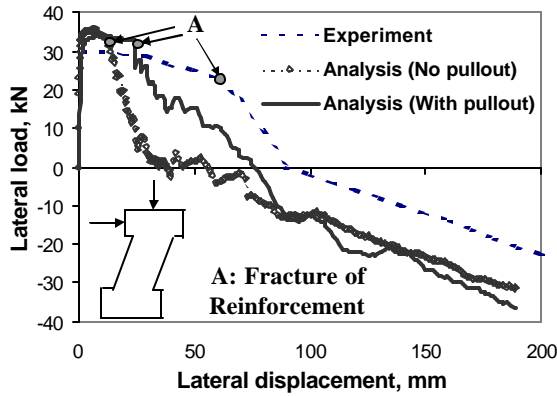


Fig.6 Analytical simulation of collapse test⁹⁾

The reinforcement fibers in compression are analyzed by average stress-strain relationship that includes stress softening in high compression due to lateral deformation of longitudinal reinforcing bars; i.e. buckling. Of course, the existence and extent of stress softening is modeled to depend on many factors such as arrangement and spacing of lateral ties, strength and size of longitudinal reinforcing bars. The axial strain of reinforcement fiber is monitored and once the plastic compressive strain representing the buckling tendency attains critical spalling strain corresponding to the present

compression level in cover concrete fibers, spalling model is activated. The summary of spalling and buckling models is reported in reference⁸⁾.

To apply the tension material models, the concrete cells in the cross section are again divided into two zones. Although concrete at the crack section cannot carry any tensile stress, concrete between the cracks still carries some tensile stress due to the bond between concrete and reinforcing bars. However, the concrete fibers far from the reinforcing bars lack the bond advantage and show rapid decrease of tensile stress in post-cracking range. Hence, the portion having no bond is separated as PL (plane concrete) zone and tension-softening model⁴⁾ is used to compute the tensile behavior of concrete in this zone. On the other hand, the concrete fibers close to longitudinal bars carry zero stress at the crack section and the stress increases due to bond effect as the distance from crack section increases. Consequently, the post-cracking tensile stress averaged within the element domain in these fibers is relatively larger than that in PL zone. Hence, the active bond zone close to reinforcement is named RC (reinforced concrete) zone and tension-stiffening model⁴⁾ is adopted for concrete fibers in this zone. An et al.³⁾ have provided an analytical method to compute the maximum limit of size of RC zone based on the

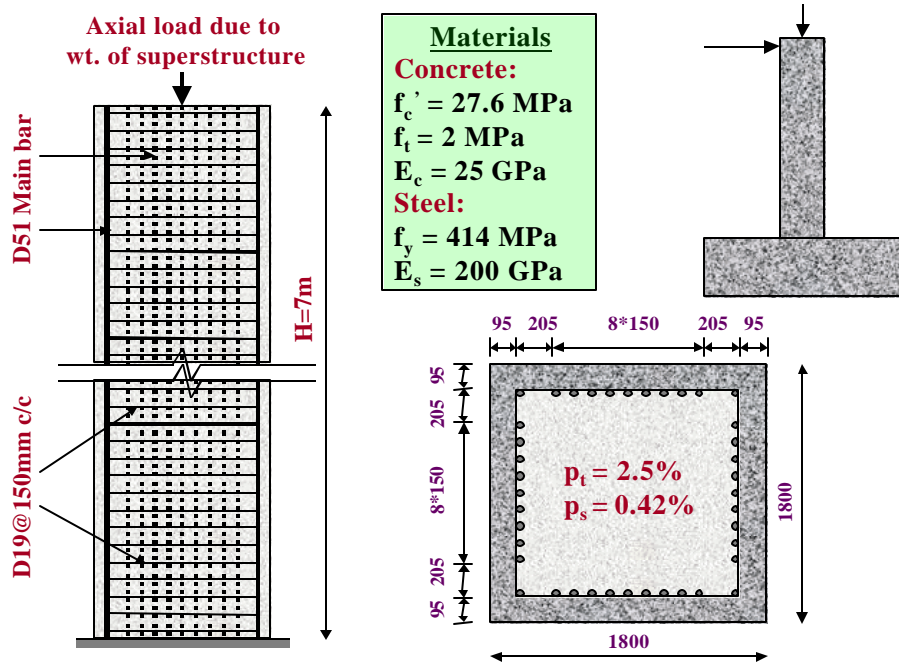


Fig.7 Target structure for FEM analysis (Unit: mm, where not given))

equilibrium between the yielding capacity of reinforcement and tensile capacity of concrete in RC zone. Nevertheless, the depth/thickness of RC zone is assumed to be equal to two times cover concrete thickness in this study. To compensate the post-cracking tensile stress in concrete, the reinforcement stress distribution along the element shows opposite nature; i.e. larger stress at the crack section and local stress decreases as the distance from crack section increases. Hence, the average stress-average strain relationship of reinforcing bar is different from the pointwise stress-strain behavior. For instance, the average stress and average strain corresponding to the first yielding are less than the local yield stress and yield strain. Hence, to include the effect of bond, average stress-strain relationship¹⁴⁾ is used for reinforcement in tension. Moreover, the steel reinforcements are modeled to break when the tensile strain reaches 20%.

To investigate the applicability of these analytical models for flexural columns in high-displacement range, one of the aforementioned experiments is simulated here. Fig.6 shows the analytical predictions of lateral load-displacement relationship with and without considering reinforcement pullout at the column footing joint. The footing is not explicitly considered in the analysis and a fixed support is provided at the base of the column. A constant compression is applied at the top of the topmost element and the total Lagrangian geometrical nonlinearity is considered in the analysis to include P-delta effect. Pullout of

reinforcing bars at the column-footing joint is taken into account by using a link element between the fixed support and the bottommost frame element, which is analyzed by exact bond pullout model¹⁵⁾. Comparing with the experimental result, it can be said that the analytical prediction with pullout is fair enough. Nevertheless, the maximum load in the analysis is slightly larger and the displacement, at which fracture of longitudinal reinforcing bars occurred (point A) along with the displacement, at which lateral load becomes zero, are slightly less than those in experiment. These differences may be because the top of the column is assumed as perfectly fixed in the analysis, whereas it is reported⁹⁾ that small rotation might have taken place in the experiment. If the restrain at the top can be slightly released in the analysis, the stiffness will decrease and point A as well as the critical displacement will obviously come closer to the experimental values. However, this comparison verifies the reliability of this analytical tool in qualitatively predicting the response of flexural columns in high deformation.

5 . RESULTS AND DISCUSSION

(1) Target Structure and Analytical Results

For detail analytical investigation to assess the probability of flexural instability, a large bridge-pier as shown in Fig.7 is considered. This pier represents the typical size bridge pier. The height of this pier is

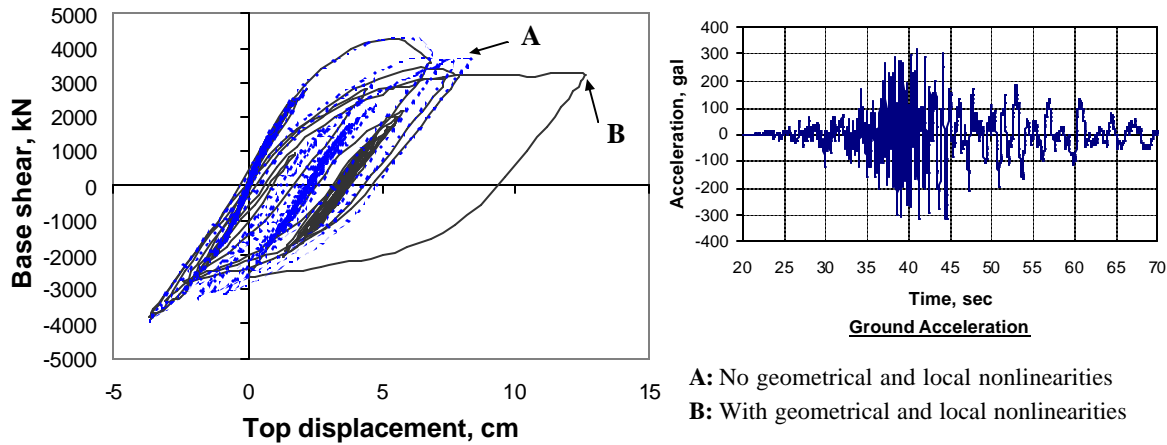


Fig.8 Effect of nonlinearities in seismic response

7m and the cross-section is 180*180cm square. Longitudinal reinforcements consist of 40 numbers of 51mm diameter steel bars (2.5% reinforcement ratio) and the transverse reinforcements are provided with 19mm diameter ties spaced at 15cm center to center. The mechanical properties of the concrete and reinforcement are also given in Fig.7. The shear capacity to bending capacity ratio is found to be slightly higher than 1.3. Hence, failure mode can be expected to be flexure. To ensure higher ductility, the strength ratio can be increased by increasing the amount of lateral reinforcements. The additional stirrups will not have much influence on the flexural behavior, except for delaying the buckling tendency of longitudinal reinforcing bars.

This pier was designed⁷⁾ according to JSCE seismic design code for a ground acceleration equivalent to elastic response of 1g and the weight of superstructure is equal to 7000kN. Here, a detail analytical study is conducted to investigate the effect of local and geometrical nonlinearity in the post-peak flexural response of the pier.

First of all, the pier is analyzed under ground motion (Fig.8). The ground motion and the axial load are similar to those used during the design of the pier. The analysis was carried out with and without considering geometrical and local nonlinearities. The difference between these two cases is shown in Fig.8.

As can be seen from the comparison, the post-peak load is slightly overestimated if the associated geometrical nonlinearities are overlooked. The analytical result also shows that the maximum and residual displacements are underestimated when the geometrical nonlinearities are neglected. This might affect the evaluation of seismic level 2 performance; i.e. the response ductility should be within allowable limit. It is to be mentioned here that only one seismic case is analyzed in this study and it is not

rational to generate any general conclusion based on the result of one case. For this purpose, a more detail investigation is required, which is out of scope of this study and should be performed in the near future. However, the analytical result of this case hints that geometrical and local nonlinearities should be considered in performance checking, although they might yield slightly conservative designs, which should be preferred ahead of those unsafe designs resulting from neglecting these nonlinearities.

Next, the same column is studied under static and monotonic lateral loading. To focus on the effect of geometrical nonlinearity, much higher axial load (20000kN) is applied at the top. Analyses are carried out with four different combinations of geometrical and local nonlinearities (cover concrete spalling and reinforcement buckling) and the predicted load-displacement relationships are presented in Fig.9.

These results can also be used to verify the mechanisms explained in Fig.4. As can be observed from the comparative analytical curves, P-delta effect and material inelastic mechanisms such as spalling and buckling play important role in the overall post-peak response of RC columns in high deformation state. Two facts can be well understood from the above comparative curves. First, the flexural capacity significantly decreases in high displacement range due to cover concrete spalling and reinforcement buckling. Second, the resisting force never becomes negative if P-delta effect is neglected.

When P-delta effect and local nonlinearity are not considered (curve A in Fig.9), the flexural response in high displacement comes from the reinforcing bars as the concrete contribution is negligible due to high compressive strain. Because of the elasto-plastic behavior, assumed in the

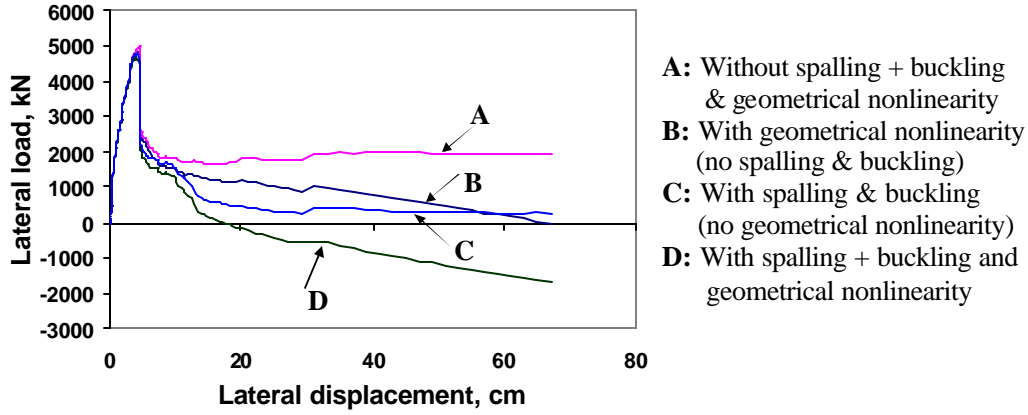


Fig.9 Effect of nonlinearities in monotonic loading

constitutive model of reinforcing bars, the high deformation response is nearly constant until the reinforcement breaks. It was found that the resisting force slightly increases in high deformation range if stress hardening of reinforcement in tension as well as compression is considered. If spalling and buckling are considered without P-delta effect (curve C in Fig.9), similar behavior can be noticed. However, the decrease of post-peak resisting force is accelerated, but it remains nearly constant at a lower positive value in the later stage. The difference between the ultimate resisting forces in these two cases comes from the decrease in flexural capacity due to reduction in the compressive stresses carried by buckled reinforcements.

In the other hand, when P-delta effect is given due consideration (curves B and D in Fig.9), the resisting force keeps on decreasing due to increase in overturning moment caused by the axial load. The effect of geometrical nonlinearity can be evaluated by subtracting the resisting force in case B from case A and again case D from case C, too. As expected, the differences in lateral loads in these two cases were found to be exactly the same, indicating that P-delta effect is external structural mechanism and is independent of material behavior. From Fig.2, it is understood that the difference in lateral load due to P-delta effect can be calculated as Pd/H , where P , d and H are axial compression, lateral displacement and column height, respectively. Here, a comparison between the analytically predicted (difference between curves A and B in Fig.9) and calculated ($P*d/H$) difference in lateral load due to P-delta effect is shown in Fig.10. These two curves are found to coincide with each other, verifying the geometrical nonlinearity considered in the analysis.

In Fig.9, it can be noticed that the negative lateral force in high displacement range can be predicted if P-delta effect is considered in analysis. Moreover, it can also be observed that this behavior

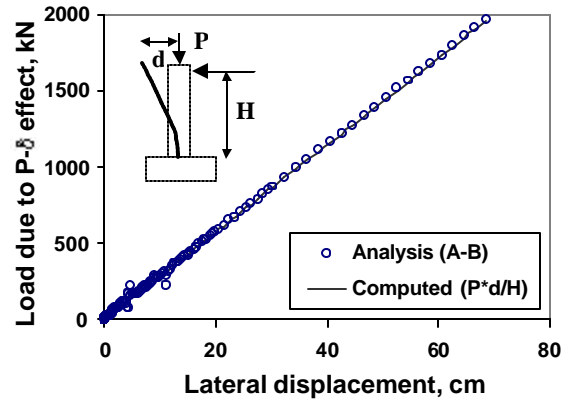


Fig.10 Difference in lateral load due to P-delta effect

is accelerated by local material nonlinearities such as spalling and buckling. As explained earlier in Fig.4, lateral load is equal to zero when the overturning moment due to P-delta effect becomes equal to the flexural capacity of the section at that deformation level. Hence, if the lateral displacement is increased beyond this critical point, predicted lateral load becomes negative. The physical meaning of this mechanism is that a lateral load in opposite direction has to be applied to keep the structure stable in this displacement state.

(2) Investigation of flexural instability

To understand the physical consequences of negative lateral restoring force, two more detail analyses are performed. Both P-delta effect and local nonlinearities are considered in the analysis (curve D in Fig.9). First, the monotonic displacement is applied at the top of the pier until the desired displacement level is achieved. Fiber analysis is carried out and the fiber strains, stresses and path dependent parameters at the last loading step are stored. Next, the lateral load at the top is released and the pier is subjected to its dead load

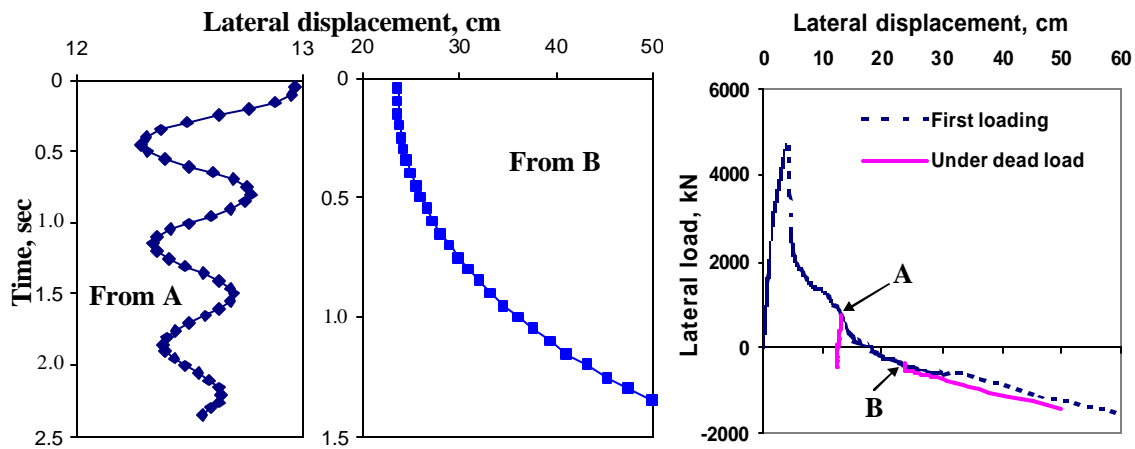


Fig.11 Stability of RC pier under dead load I high displacement range

and the weight of the superstructure only. The path dependent parameters stored in the previous run are used as the initial conditions for this analysis. The restart function included in the analytical tool enables to perform such analyses. In the second run, dynamic fiber analysis is performed with zero ground accelerations in X and Y directions in horizontal plane and acceleration equivalent to gravity is applied in the vertical direction to account for the effect of dead loads (self weight and the weight of superstructure). To simulate the inertia force, the superstructure is modeled as a concentrated mass instead of a constant axial load at the top of the pier. Two sets of analyses, in which the second stage loading starts at displacements respectively smaller and larger than the critical displacement corresponding to zero lateral load (points A and B in Fig.11), are discussed here. The analytical results are illustrated in Fig.11.

Some interesting behaviors could be discovered through these analyses. When the pier is allowed to deform freely at a displacement smaller than the critical displacement (point A in Fig.11), it undergoes free vibration and the pier can reach a stable state with some residual displacement after some time. In contrast, if the pier is allowed to deform freely at a displacement larger than the critical displacement (point B in Fig.11), the lateral displacement keeps on increasing with time, indicating that the pier is unstable. These analytical results give ample evidence that RC piers under axial load can collapse due to structural instability if the residual displacement after an earthquake is higher than critical displacement. It can also be imagined that even if the residual displacement after a major earthquake is less than the critical displacement, the concerned structure can become unstable due to relatively smaller aftershocks.

6 .PARAMETRIC STUDY

As mentioned earlier, the effect of geometrical nonlinearity increases with increase in deformation level whereas the flexural capacity of RC piers decreases due to compression softening of concrete, cover concrete spalling as well as buckling and breaking of longitudinal reinforcements. Once the critical deformation is reached, the overturning P delta moment becomes equal to the section capacity. Hence, it can be argued that critical displacement depends on the axial load, consisting of the self-weight of pier and the weight of superstructure, as well as the bending capacity of the section, which is mainly governed by amount of main reinforcement provided the cross-section and material properties are unchanged. Here, a parametric analysis is performed with different axial loads and reinforcement ratio.

(1) Effect of axial load

Usually in bridge piers, the top mass is less than 10% of the axial capacity of the piers but RC columns inside a building might be subjected to relatively higher axial load due to heavy slab at the top. Recently, construction of very high piers is becoming common in Japan. Because of their long height and huge cross-section, the self-weight of these piers often outstrip the weight of superstructure. Obviously, the effect of geometrical nonlinearity will be more significant in such cases. For clear understanding, the aforementioned pier is analyzed under lateral displacement with different levels of axial load ranging from 0 to 22% of the axial capacity. The axial load is applied statically at the top of the pier and the self-weight is not considered in the static analysis. The load-

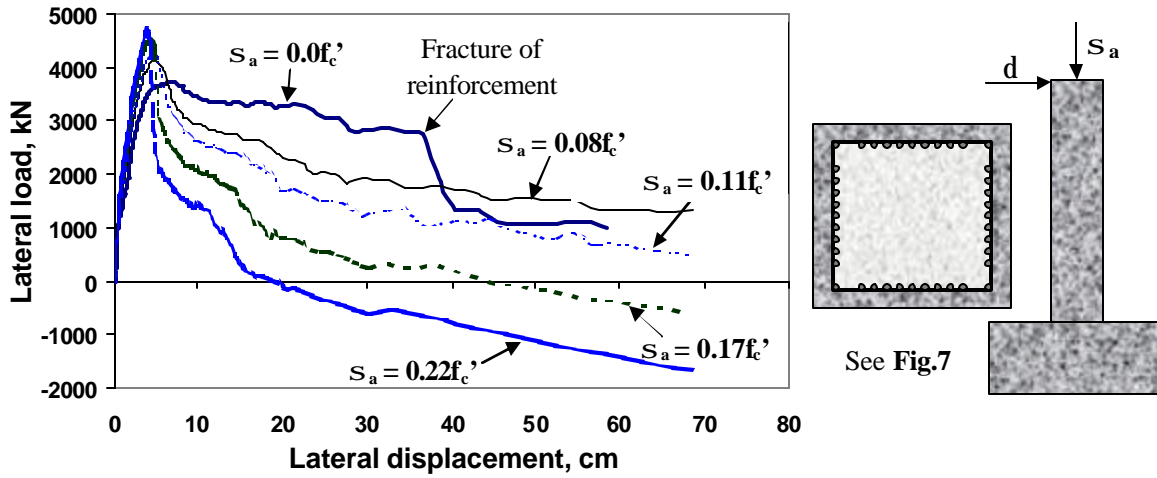


Fig.12 Effect of axial load in collapse mechanism

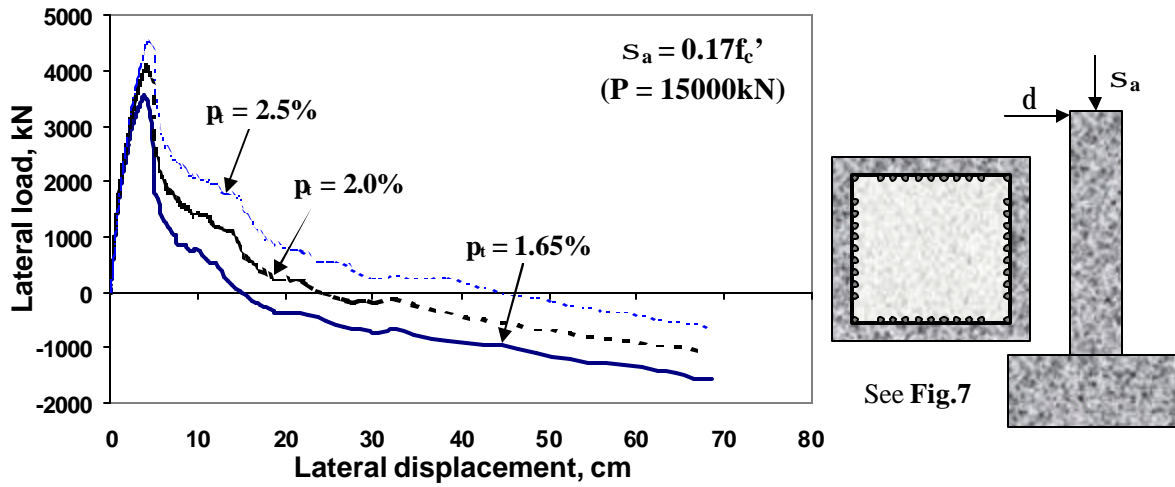


Fig.13 Effect of reinforcement ratio in collapse mechanism

displacement relationships predicted through analysis are shown in Fig.12.

As expected, the pre-peak stiffness and peak load show slight increase with increase in axial load. When axial load is increased, higher uniform compressive strain is developed throughout the cross-section and tensile mechanisms like cracking, yielding and breaking of reinforcement are delayed. In contrast, compression inelastic mechanisms such as cover spalling and reinforcement buckling occur earlier and the post-peak softening phenomenon becomes more prominent with increase in axial load. Consequently, the critical displacement, after which the pier becomes unstable, also decreases. Hence, the flexural instability might become predominant in case of high RC piers with large section due to the combined effect of self-weight and the overlying top mass.

(2) Effect of reinforcement ratio

It is obvious that reduction in the amount of main reinforcement decreases the bending capacity whereas the shear capacity is nearly unaffected. Consequently, the shear to bending capacity ratio increases and according to the current design code, it can be argued that the structure becomes less liable to complete collapse and higher ductility can be ensured. In other words, the provisions in current JSCE seismic design code lead to the argument that reduction of main reinforcement renders RC pier safer against complete collapse in spite of higher residual displacement after an earthquake. Of course, the reduction of main reinforcement ensures that shear failure cannot occur even in higher deformation range. But, the authors feel it necessary to answer one more question: Can a reinforced concrete pier with small amount of main

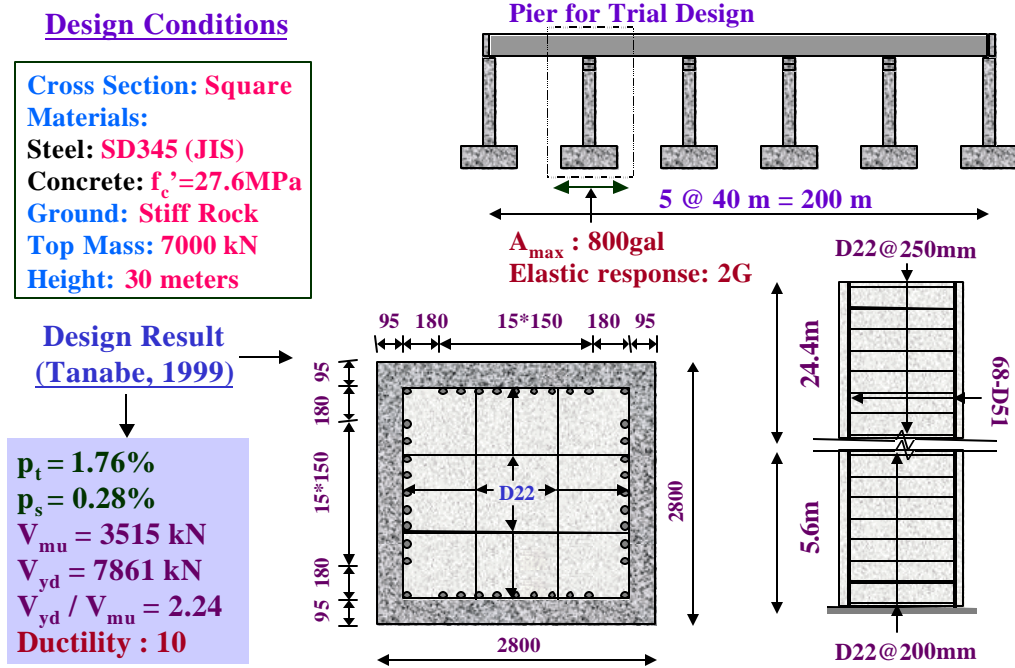


Fig.14 Long pier designed by JSCE seismic design code (Unit: mm, where not given)

reinforcement maintain stability in high deformation range?

To answer this question, the aforementioned pier is analyzed with different reinforcement ratio. Lateral displacement is monotonically applied at the top of the pier with constant axial load equal to 15000kN applied at the top. The lateral load-displacement curves obtained from analysis are illustrated in Fig.13. As expected, the peak load and the yielding displacement become smaller with decrease in reinforcement ratio. It can be noticed that the post-peak softening is significantly influenced by the reinforcement ratio. Interestingly, it was observed that a small decrease in reinforcement ratio causes significant reduction of the critical displacement. It can be noticed that if reinforcement ratio of the pier is changed from 2.5% to 2%, the critical displacement is nearly halved. The critical displacement defines the deformation range until which the structure is stable. Hence, in spite of the fact that shear collapse can be avoided by reducing the amount of main reinforcement, such structures are liable to earlier collapse due to flexural instability.

7. CASE STUDY

Based on earlier discussions, it has become clear that RC piers, in some cases, might become unstable due to geometrical nonlinearity associated with high

axial load. This possibility is prominent especially in high RC piers with comparatively less reinforcement. In such high piers, the bending capacity is considerably smaller than the shear capacity and shear failure can be easily avoided. According to JSCE design code, high ductility ratio equal to 10 can be ensured for such piers without any risk of collapse. But as the code says, geometrical nonlinearity has to be considered as well, if necessary.

For further clarification, a long pier designed⁷⁾ according to JSCE seismic design code is studied in detail. This design was carried out for a normal highway bridge pier in Japan with span of around 40m and deck-width of around 10m, the weight of superstructure being 7000kN. The 30m long pier was designed for a ground motion with maximum ground acceleration of 800 gal, the elastic response being 2g. Concrete with characteristic compressive strength equal to 24MPa and steel (SD345) with characteristic yield strength equal to 345MPa were considered in design. In the examination of the ultimate limit state, the material strengths are considered to be 1.15 and 1.2 times of the characteristic value for concrete and steel, respectively. The design conditions as well as the design details are shown in Fig.14.

The detailed design yielded a square cross-section (280*280cm) with symmetrically arranged 68 longitudinal reinforcing bars with 51mm diameter and four-legged 22mm diameter stirrups

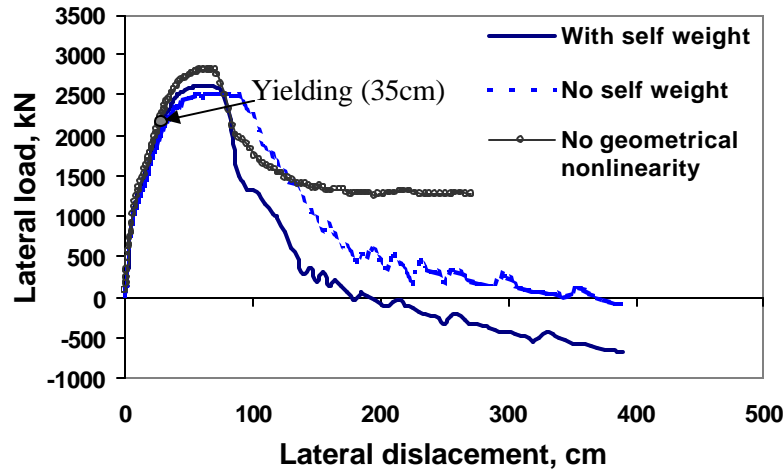


Fig.15 Effect of self weight in collapse of long pier

spaced at 20cm center to center. The longitudinal and transverse reinforcement ratios are 1.76% and 0.28%, respectively. The shear to bending strength ratio is 2.24, thus ensuring ductility ratio equal to 10 according to the design code.

This pier is analyzed with monotonic lateral displacement applied at the top of the pier. The top mass of the deck is simulated by a constant axial load equal to 7000kN. Because of high length and huge cross-section, the self-weight of the piers amounts to be more than 80% of the weight of superstructure. Hence, the analysis was carried out twice, with and without considering self-weight of the pier. The result of push-over analysis is shown in Fig.15.

As expected, the responses of the pier, predicted from these two analyses, are considerably different. In both cases, the reinforcements yielded when the applied top displacement was around 35cm. When self-weight is overlooked, the pier could be loaded, without impairing its stability, until the response displacement is about 10 times yielding displacement. However, giving due consideration to the self-weight of this pier caused significant reduction in the critical displacement. It can be noticed from the figure that the pier becomes unstable once the response displacement exceeds 5 times yielding displacement. It suggests that the JSCE seismic design code statement “A ductility factor m_f of approximately 10 can be ensured if V_{yd}/V_{mu} is equal to or larger than 2 and no special consideration is needed” is acceptable for cases without much geometrical nonlinearity. In contrast, if the effect of geometrical nonlinearity cannot be ignored, it is needed to reduce the allowable inelastic response attributed to the P-delta effect. To avoid the flexural instability due to geometrical

nonlinearity, a further checking is necessary before deciding the allowable ductility.

8 . CONCLUSION

From the extensive analytical study on highly inelastic lateral response of axially loaded reinforced concrete piers, the following conclusions can be drawn. The ultimate aim of seismic design is to avoid fatality by ensuring that the structure does not collapse even after facing significantly high damage. This goal can be achieved if the following two conditions are fulfilled.

First, because of its brittleness and fatality, shear failure should be completely avoided no matter how large the damage level is. According to the current JSCE seismic design code, this condition can be satisfied if the shear strength is higher than bending strength of the designed structure. A small amendment in this recommendation is felt necessary to incorporate the degradation of shear contribution of concrete with increase in damage level.

Second, the structure should be geometrically stable within allowable displacement range, as the code states. Through extensive analysis, the existence of collapse mechanism even in flexure, led by instability due to geometrical nonlinearity associated with high axial load, is proved. The parametric study revealed that this behavior is accelerated by local nonlinearities such as cover concrete spalling and reinforcement buckling etc. and the range of lateral displacement, throughout which the structure is stable, decreases with increase in axial load and decrease in the amount of longitudinal reinforcement. Analysis of a long pier designed according to JSCE seismic design code showed that flexural instability could occur much

before the allowable ductility when no P-delta effect exists. Hence, it is highly recommended that geometrical and local nonlinearity be considered either explicitly or implicitly in deciding the allowable ductility of RC piers of larger height and with heavier top mass.

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